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LEVEL II

TECHNIQUE FOR ESTIMATING THE MAGNITUDE AND FREQUENCY OF FLOODS IN THE HOUSTON, TEXAS, METROPOLITAN AREA

AD A089495

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations 80-17

Prepared in cooperation with the City of Houston, the Harris County Flood Control District, the Texas Department of Water Resources, and the U.S. Army Corps of Engineers

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A technique for estimating the magnitude and frequency of floods in the Houston, Texas, metropolitan area was developed by use of a multiple-regression flood-frequency analysis of flow data from unregulated streams in the area. A regression model, relating flood-peak discharge to concurrent rainfall and antecedent soil moisture conditions, was used to simulate 67-year records of annual peak discharges. Flood-frequency characteristics were determined for the simulated annual peaks and for the observed annual peaks at each of 22 gaging stations. Drainage area, bank-full channel conveyance, and percentage of urban development were used as independent variables; and weighted flood-frequency discharges were used as dependent variables in the multiple-regression analysis.

Relationships applicable to unregulated streams were developed for predicting floods with recurrence intervals of 2, 5, 10, 25, 50, 100, and 500 years. Drainage basins ranged in area from 1.33 to 182 square miles. The percentage of urban development in these basins ranges from 37 to 98.9 percent.

The relationships indicate that as a basin changes from a completely natural state to one of complete urbanization, the magnitude of a 2-year peak discharge is increased by a factor of 4.2, the magnitude of a 50-year peak is increased by a factor of 4.9, and the magnitude of a 100-year peak is increased by a factor of 4.9.

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Cover photograph, Whiteoak Bayou in flood as it approaches the downtown area, courtesy of the Harris County Flood Control District

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(10)

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By Fred Liscum [redacted] B.C. Massey

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UNITED STATES DEPARTMENT OF THE INTERIOR

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TECHNIQUE FOR ESTIMATING THE MAGNITUDE
AND FREQUENCY OF FLOODS IN THE
HOUSTON, TEXAS, METROPOLITAN AREA

By
Fred Liscum and B. C. Massey
U.S. Geological Survey

ABSTRACT

A technique for estimating the magnitude and frequency of floods in the Houston, Texas, metropolitan area was developed by use of a multiple-regression flood-frequency analysis of flow data from unregulated streams in the area. A regression model, relating flood-peak discharge to concurrent rainfall and antecedent soil moisture conditions, was used to simulate 67-year records of annual peak discharges. Flood-frequency characteristics were determined for the simulated annual peaks and for the observed annual peaks at each of 22 gaging stations. Drainage area, bank-full channel conveyance, and percentage of urban development were used as independent variables; and weighted flood-frequency discharges were used as dependent variables in the multiple regression analysis.

Relationships applicable to unregulated streams were developed for predicting floods with recurrence intervals of 2, 5, 10, 25, 50, 100, and 500 years. Drainage basins ranged in area from 1.33 to 182 square miles. The percentage of urban development in these basins ranges from 37 to 98.9 percent.

The relationships indicate that as a basin changes from a completely natural state to one of complete urbanization, the magnitude of a 2-year peak discharge is increased by a factor of 4.2, the magnitude of a 50-year peak is increased by a factor of 4.9, and the magnitude of a 100-year peak is increased by a factor of 4.9.

INTRODUCTION

Purpose of This Report

In 1964, the U.S. Geological Survey, in cooperation with the city of Houston, began a program to define the effects of urbanization on flood characteristics in the Houston, Texas, metropolitan area. Such information is necessary for the proper design of flood-plain structures, for flood-plain management, and for the determination of flood-insurance rates.

An earlier report by Johnson and Sayre (1973) presented a technique for estimating the magnitude of flood-peak discharges for recurrence intervals of 2 to 100 years. Johnson and Sayre (1973) developed equations that related the flood-peak discharge for a particular recurrence interval to the area of the drainage basin and to the degree of urbanization.

This report was prepared in cooperation with the city of Houston, the Harris County Flood Control District, the Texas Department of Water Resources, and the U.S. Army Corps of Engineers. It presents a technique similar to that of Johnson and Sayre (1973) and should be used in preference to it. The technique should provide reliable estimates of the magnitude of floods with recurrence intervals of 2, 5, 10, 25, 50, and 100 years for unregulated streams in the Houston metropolitan area.

The reliability of flood-frequency estimates for very large recurrence intervals is uncertain; therefore, values for the 500-year flood are omitted from this report. However, an equation is provided primarily for the use of planners who are required to compute the magnitude of a 500-year flood for special purposes such as flood-insurance studies.

In the development of this technique, the observed annual flood peaks were compiled for each of 22 gaging stations. In addition, a digital model was used to simulate a 67-year record for annual peak discharges for each of the 22 sites. Standard statistical methods recommended by the U.S. Water Resources Council (1977) were used to obtain two sets of flood-frequency discharges for each site. The two sets of values were then combined to obtain a single flood-frequency curve for each site.

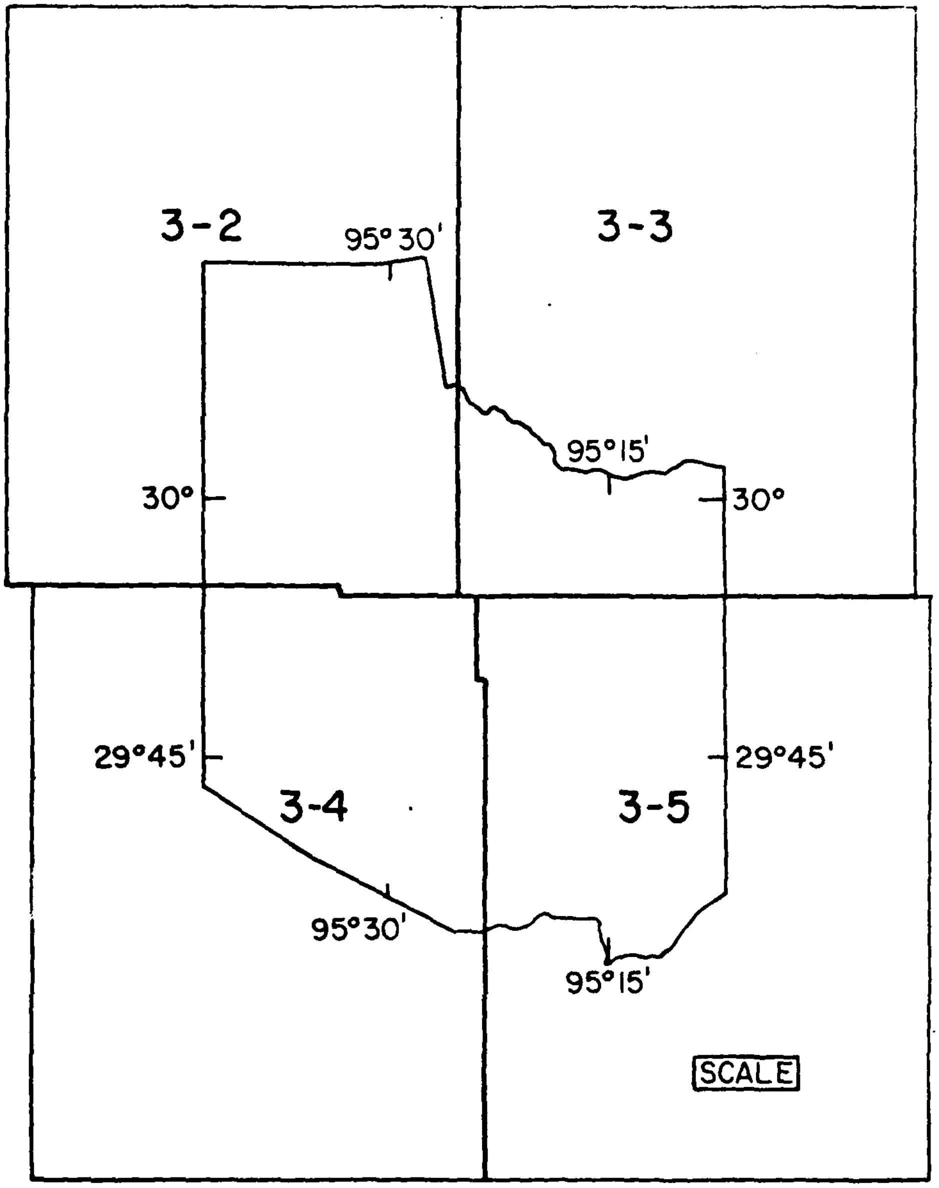
Johnson and Sayre (1973) considered 12 basin characteristics to define the variation in flood-peak magnitudes in the Houston metropolitan area. Only two characteristics, size of drainage area (A), and a measure of urban development (percent impervious, I) were selected as useful. Johnson and Sayre also recognized the importance of sufficient channel capacity. The results of their report were considered in the selection of basin characteristics for this study. Multiple-regression techniques were used to define the relationships between flood-peak magnitude and selected basin characteristics.

Description of the Area

The Houston metropolitan area (fig. 1), which encompasses about 1,000 square miles, is located on a flat coastal plain about 45 miles from the Gulf of Mexico. The soils are predominantly clays, but vary from fine sandy loams in the northern part of the area to heavier clay loams south of Buffalo Bayou.

FIGURE 1

STUDY AREA AND LOCATIONS OF DATA-COLLECTION SITES

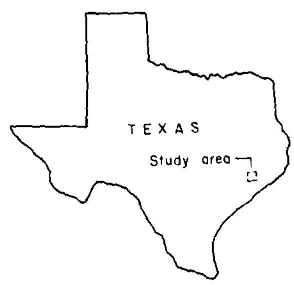


Index map showing page numbers of each component of figure 1

1

EXPLANATION

- ▲ Stream-gaging station
- ▼ Water-quality sampling site
- △ Flood-hydrograph partial-record station
- ◭ Low flow partial-record station
- ● Recording rain gage
- ○ Nonrecording rain gage



Location map

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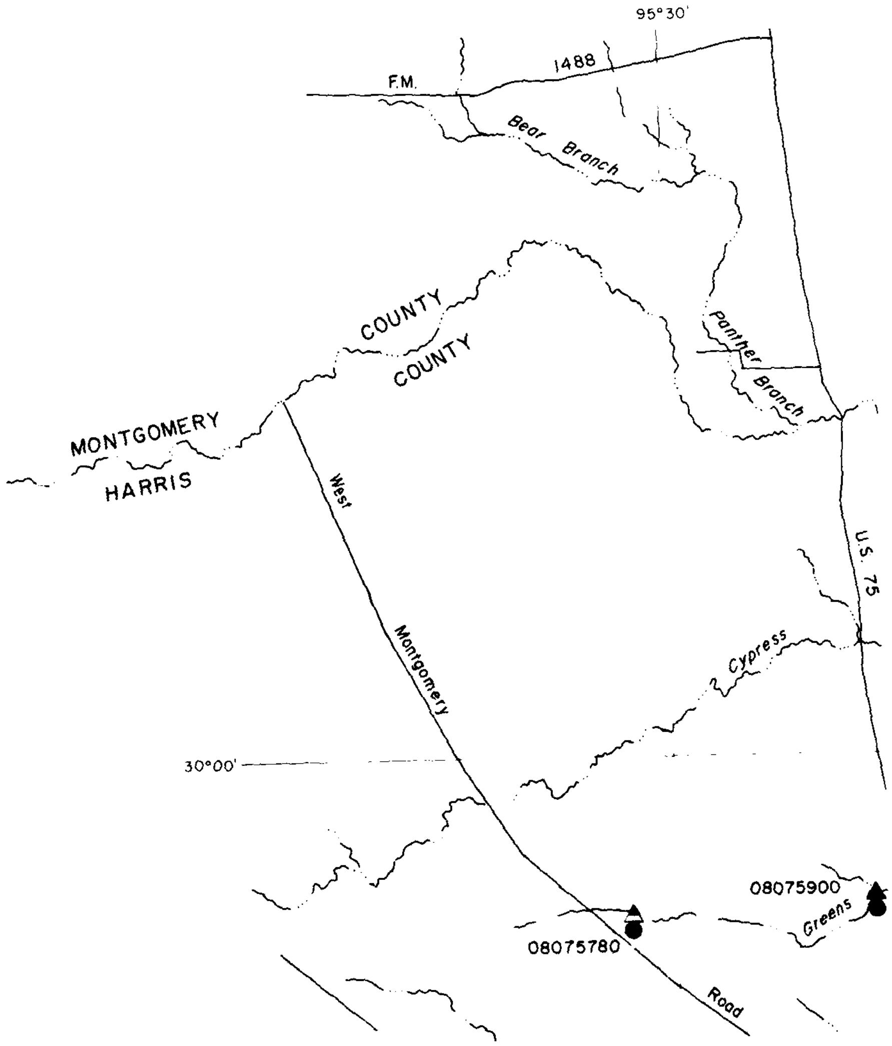

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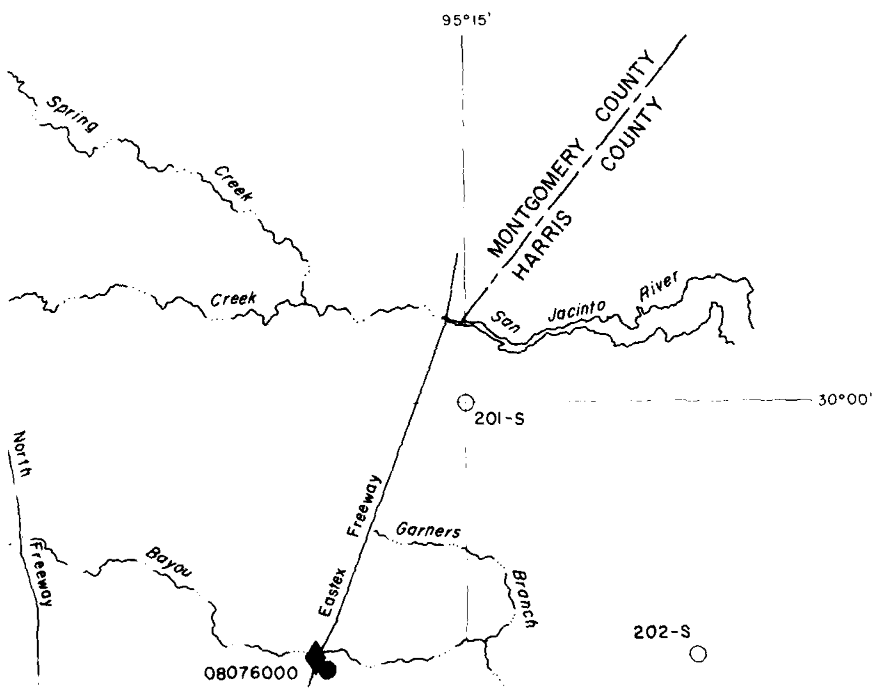
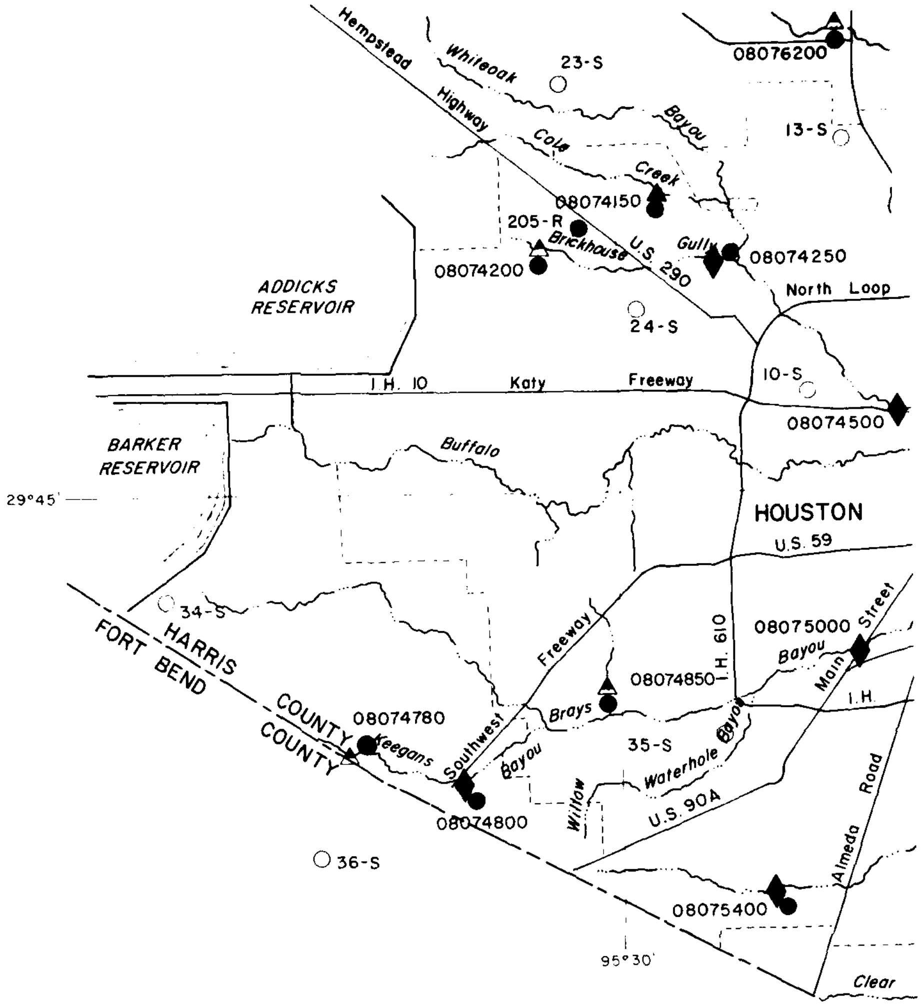
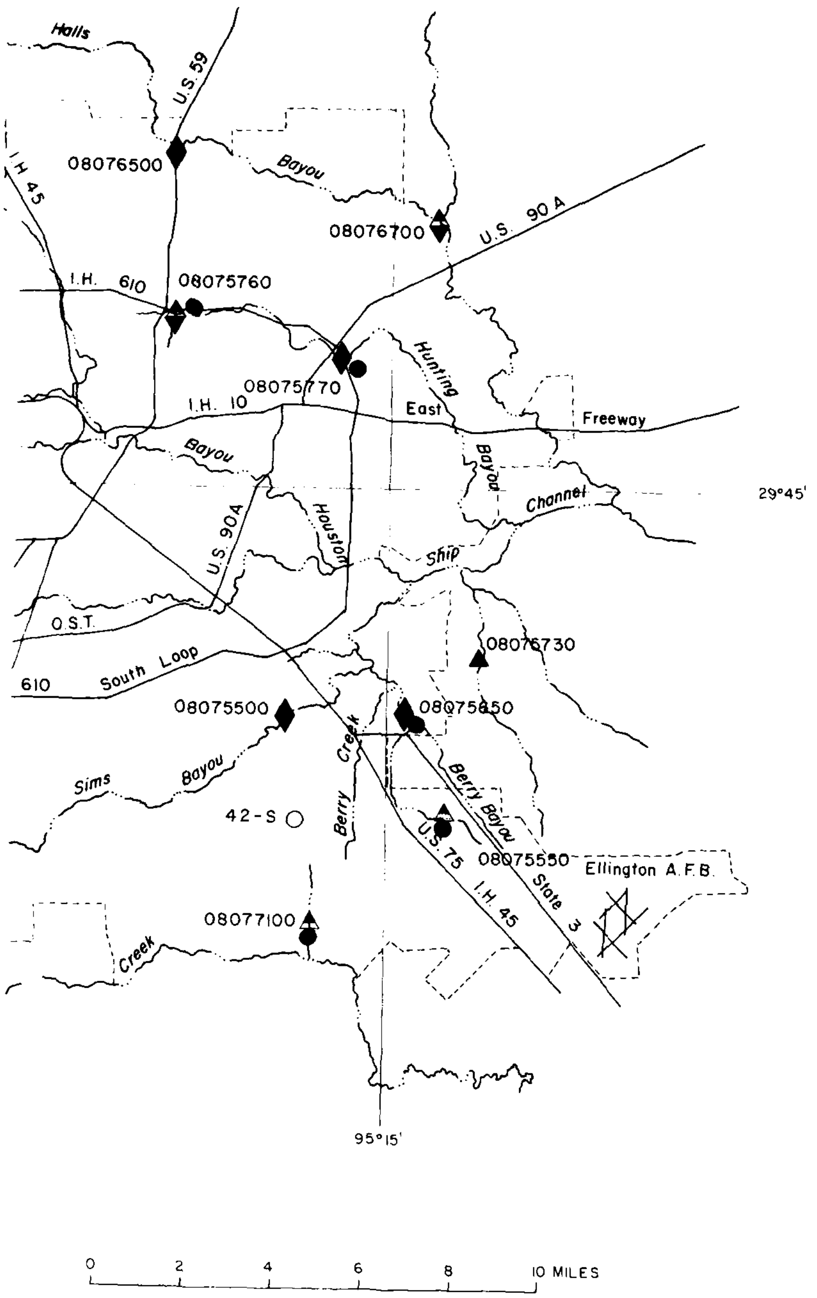




Base from Texas General Highway Map



Halls
U.S. 59
08076500
Bayou
I.H. 45
08076700
U.S. 90 A
I.H. 610
08075760
Hunting
08075770
I.H. 10
East
Freeway
Bayou
Bayou
Channel
29°45'
Houston
U.S. 90A
Ship
O.S.T.
610 South Loop
08075730
08075500
08075850
Sims
Bayou
Creek
Berry Bayou
42-S
Berry
U.S. 75
08075550
Ellington A.F.B.
State 3
I.H. 45
08077100
Creek
95°15'
0
2
4
6
8
10 MILES

The climate is characterized by short, mild winters; long, hot summers; high relative humidity; and prevailing southeasterly winds. The mean annual temperature (1941-70) is 68.9°F (20.5°C). The 30-year average (1941-70) rainfall for Houston is 48.19 inches, which is distributed fairly uniformly throughout the year.

The major stream draining the Houston area is Buffalo Bayou, a tributary to the San Jacinto River. Buffalo Bayou is regulated by the Barker and Addicks flood-detention reservoirs near the western limits of the area. From these reservoirs, Buffalo Bayou meanders eastward to the Houston Ship Channel, and along its course, is fed by five major tributaries: Whiteoak, Brays, Sims, Hunting, and Greens Bayous.

The channel-bed slopes (3 to 8 feet per mile) are relatively flat and few of the drainage-basin divides are defined accurately by natural features. Basin exchange, which is runoff to or from an adjacent basin, often results from heavy rainfall; and in many places, adjacent basins are interconnected by ditches to relieve poorly drained areas. All of the major stream channels have been improved.

Metric Conversions

For readers interested in using the metric system, the inch-pound units used in this report may be converted to metric units by the following factors:

From		Multiply by	To obtain	
Unit	Abbreviation		Unit	Abbreviation
cubic foot per second	ft^3/s	0.02832	cubic meter per second	m^3/s
foot	--	0.3048	meter	m
foot squared	--	0.0929	meter squared	m^2
foot per mile	ft/mi	0.189	meter per kilometer	m/km
inch	--	25.4	millimeter	mm
square mile	--	2.590	square kilometer	km^2

AVAILABILITY OF DATA

Data from 22 gaging stations were used in this analysis. Thirteen of the stations are equipped with continuous recorders; nine are equipped with flood-hydrograph recorders that record water-surface elevations only during times of storm runoff. Sixteen of these stations are instrumented to obtain a continuous rainfall record. Another 25 recording rain gages and 10 nonrecording gages are located at sites other than gaging stations. In addition, 67 years of 5-minute rainfall data (1910-76) are available from the National Weather Service station at Houston.

Annual Peak Discharges
Observed Data

The locations of the 22 gaging stations used in this study are shown on figure 1. The period of record for each station is given in table 1. The annual-peak discharges used in the analysis of observed data are given in table 2. Because the influence of channel capacity on the magnitude of annual peak discharges in the Houston area is greater than that of other indices of urbanization, the period of record for this analysis was selected as the period since the last major channel rectification at each of the sites.

None of the peak discharges presented in table 2 resulted from tropical storms or hurricanes. At some streamflow sites in central and south-central Texas, extremely high discharges have resulted from the intense rainfall associated with such storms. Data from these sites indicate that tropical storms may have the potential to produce flood discharges greatly in excess of those estimated when using the relationships developed in this study.

Simulated Data

A long-term record of flood peaks was simulated for each site by using a regression model developed by Johnson and Sayre (1973). The flood-frequency estimates determined from these synthetic data were combined with those determined from observed data to provide more reliable estimates of flood-peak discharges for the selected recurrence intervals.

The regression model related observed flood-peak discharges to concurrent rainfall and antecedent conditions. It has the following form:

$$Q_p = aP^{b_1}D^{b_2}M^{b_3}$$

where Q_p = peak discharge, in cubic feet per second;

P = the Theissen-weighted storm rainfall, in inches;

D = storm duration, in hours, during which 85 percent of the rainfall (P) occurred;

M = the soil-moisture index as defined below;

a = the regression constant; and

b_1, b_2, b_3 = regression coefficients.

The soil-moisture index, M, was defined by Johnson and Sayre (1973) as

$$M = (M_0 + P_0)\, k^t$$

where M = soil-moisture index, in inches, for the day on which the peak discharge occurred;

M_0 = the last computed soil-moisture index, in inches, for t days preceding the storm;

Table 1.-- Period of record for gaging stations used in the flood-frequency analysis

Sequence	Station	Station name	Period of record
1	08074150	Cole Creek at Deihl Rd., Houston	1964-76
2	08074200	Brickhouse Gully at Clarblak St., Houston	1964-76
3	08074250	Brickhouse Gully at Costa Rica St., Houston	1964-76
4	08074500	Whiteoak Bayou at Houston	1936-76
5	08074780	Keegans Bayou at Keegans Rd., Houston	1965-71, 1975-76
6	08074800	Keegans Bayou at Roark Rd., Houston	1964-76
7	08074850	Bintliff Ditch at Bissonnet St., Houston	1968-76
8	08075000	Brays Bayou at Houston	1936-76
9	08075400	Sims Bayou at Hiram Clark St., Houston	1964-76
10	08075500	Sims Bayou at Houston	1952-76
11	08075550	Berry Bayou at Gilpin St., Houston	1965-76
12	08075650	Berry Bayou at Forest Oaks St., Houston	1964-76
13	08075730	Vince Bayou at Pasadena	1972-76
14	08075760	Hunting Bayou at Falls St., Houston	1964-76
15	08075770	Hunting Bayou at IH-610, Houston	1964-76
16	08075780	Greens Bayou at Cutten Rd., Houston	1965-76
17	08075900	Greens Bayou at U.S. Highway 75, Houston	1965-76
18	08076000	Greens Bayou near Houston	1952-76
19	08076200	Halls Bayou at Deertrail St., Houston	1965-76
20	08076500	Halls Bayou at Houston	1952-76
21	08076700	Greens Bayou at Ley Rd., Houston	1972-76
22	08077100	Clear Creek tributary at Hall Rd., Houston	1965-76

Table 2.--Observed annual peaks used in the analysis of observed data

Station	Water year 1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976
08074150	--	--	--	--	400	338	950	160	810	966	453	762	2,020	1,790	489	1,100	1,000
08074200	--	--	--	--	--	54	121	73	328	294	169	314	399	412	163	230	240
08074250	--	--	--	--	--	550	1,040	323	2,280	1,370	925	2,800	5,800	3,600	2,640	3,520	2,830
08074500	--	--	--	--	--	--	8,320	3,330	9,120	8,760	3,750	10,600	17,300	13,600	6,060	6,180	8,480
08074780	--	--	--	--	--	94	206	59	192	136	128	201	[1]	[1]	[1]	622	485
08074800	--	--	--	--	--	140	588	43	352	659	547	751	1,060	1,570	984	966	785
08074850	--	--	--	--	--	--	--	--	1,030	968	808	1,120	930	1,130	1,040	1,050	1,170
08075000	--	--	--	--	--	3,160	9,400	4,730	12,000	9,240	11,500	15,500	11,700	24,800	8,660	18,000	29,000
08075400	--	--	--	--	--	960	2,280	350	2,200	2,280	2,320	2,230	2,020	4,220	1,360	2,830	4,500
08075500	--	--	--	--	--	3,800	6,700	1,260	4,680	7,720	8,800	5,750	3,930	10,000	3,720	11,200	11,200
08075550	--	--	--	--	--	290	607	235	738	535	285	339	362	658	472	454	--
08075650	--	--	--	--	350	800	2,630	886	3,110	1,410	816	1,540	1,530	4,500	1,210	5,080	1,570
08075730	--	--	--	--	--	--	--	--	--	--	--	--	1,730	3,360	2,630	2,490	1,440
08075760	--	--	--	--	--	236	485	399	445	380	377	666	660	778	284	359	553
08075770	--	--	--	--	166	355	1,150	920	1,460	1,050	880	2,260	3,130	3,380	830	890	2,520
08075780	--	--	--	--	--	151	514	468	390	508	268	318	190	520	282	301	223
08075900	--	--	--	--	--	--	2,730	2,060	1,140	2,600	1,670	1,860	2,940	2,860	1,580	2,580	1,750
08076000	--	--	--	--	--	--	5,360	2,540	2,580	4,000	3,600	3,000	6,500	5,000	2,590	3,390	7,730
08076200	--	--	--	--	--	130	614	710	318	596	618	451	1,180	992	510	856	905
08076500	2,230	3,400	2,540	1,870	1,470	1,250	2,640	1,110	2,340	2,560	2,340	2,300	3,780	3,550	1,720	1,940	2,460
08076700	--	--	--	--	--	--	--	--	--	--	--	--	14,100	16,700	5,980	5,300	10,400
08077100	--	--	--	--	--	100	150	132	390	294	450	291	249	400	323	203	170

[1]Station not in operation.

P_0 = the precipitation, in inches, on the day of the last computed soil-moisture index;

k = soil-moisture depletion factor; and

t = number of days between the storm and the day of the last computed soil-moisture index.

For a more detailed explanation of these variables, see Johnson and Sayre (1973).

Values for the independent variables used in the model were determined from observed streamflow and rainfall records. The period of record used in calibrating the model for each of the 22 sites was the same as that for which annual peak data were analyzed. Because the smaller peak discharges are poorly related to the weighted rainfall, only the larger peaks were used in defining the rainfall-runoff relationships.

Multiple regression techniques were used to calibrate the regression model. The calibration defined the regression constant and the parameter coefficients for each site (table 3). The standard error of estimate, Se, for the calibrated model ranges from 14.6 to 41.4 percent with a mean of 25.4 percent.

Use of the model to simulate annual-peak discharges required the use of the National Weather Service rainfall record for 1910-76 for Houston. It was assumed that this record of precipitation was representative statistically of any site within the metropolitan area, and this assumption was validated by the use of the standard statistical test of Kolmogorov-Smirnov (Ostle, 1966) for goodness of fit between two data series. The test considered rainfall data over the period of calibration for both the National Weather Service gage in Houston and various representative calibration sites.

All storms that could have produced the maximum annual discharge were selected from the rainfall records. Values for rainfall amounts, storm duration, and soil-moisture index were determined for each of these storms. Because the rainfall amounts in the long-term record reflect point values, they were adjusted on the basis of drainage-area size and storm duration to reflect basin-wide averages for each site. This technique is described in U.S. Weather Bureau Technical Paper No. 40, "Rainfall Frequency Atlas of the United States" (1961, p. 6).

Peak discharges were computed for each selected storm by using the calibrated model for each site (table 3). Finally, the 67 annual maximum discharges were selected for each of the sites from the simulated peaks (table 4).

Basin Characteristics

The effects of urbanization on streamflow in the Houston metropolitan area can be attributed to two main changes within a drainage basin. First, natural soils and vegetal cover are replaced by impervious cover due to the

Table 3.--Summary of calibration for regression model

$$Q_p = a(P^{b_1})(D^{b_2})(M^{b_3})$$

Sequence number	Station number	Constant (a)	Exponent for P (b_1)	Exponent for D (b_2)	Exponent for M (b_3)	S_e (percent)	MCC[1]
1	08074150	244.3	1.382	-0.287	0.080	16.6	0.961
2	08074200	90.6	.980	-.151	.083	18.8	.912
3	08074250	1,165.8	.987	-.246	.118	16.7	.919
4	08074500	2,306.8	.950	--	.094	27.0	.857
5	08074780	38.4	1.450	-.145	.246	25.8	.867
6	08074800	323.6	.924	-.245	--	41.4	.672
7	08074850	490.9	.758	-.156	.035	14.8	.889
8	08075000	5,164.2	1.017	-.197	.091	17.1	.942
9	08075400	712.8	.982	-.121	.087	26.8	.800
10	08075500	1,678.8	1.220	-.245	.177	27.0	.899
11	08075550	258.2	.717	-.200	.089	23.0	.756
12	08075650	704.7	1.205	-.294	.092	27.7	.865
13	08075730	1,188.5	.780	-.168	.234	14.6	.902
14	08075760	183.2	.990	-.230	--	21.3	.835
15	08075770	463.4	1.214	-.195	.199	28.9	.834
16	08075780	98.0	1.160	-.118	.241	27.0	.831
17	08075900	590.2	1.185	-.161	.210	32.3	.815
18	08076000	1,227.4	1.156	-.240	.117	26.3	.846
19	08076200	246.6	.799	-.102	--	30.6	.719
20	08076500	839.5	.815	-.122	--	22.3	.832
21	08076700	2,322.7	1.052	-.141	.084	31.3	.820
22	08077100	128.2	.723	-.130	.100	34.3	.675

[1]Multiple correlation coefficient.

Example: Station 08074150, $Q = 244.3\ P^{1.382}\ D^{-0.287}\ M^{0.080}$

Table 4.--Annual maximum discharges as determined from the simulated peaks

Water year	Station 08074150	08074200	08074250	08074500	08074780	08074800	08074850	08075000	08075400	08075500	08075550
1910	972	228	2,396	4,568	93	971	1,100	10,257	1,667	3,091	438
1911	766	232	2,493	7,301	191	642	919	11,437	1,910	4,507	470
1912	1,323	330	3,663	8,937	291	915	1,251	16,318	2,628	6,888	582
1913	1,152	261	2,677	5,561	116	1,074	1,214	12,087	1,934	3,706	474
1914	1,310	356	3,264	13,559	358	944	1,246	17,434	3,062	6,895	547
1915	2,073	464	4,850	14,178	499	1,207	1,595	23,394	3,813	10,253	698
1916	863	207	2,377	4,874	88	904	1,031	8,913	1,444	2,808	450
1917	485	147	1,630	3,239	59	556	744	6,476	1,076	2,013	336
1918	1,154	263	2,767	5,550	121	1,061	1,205	11,968	1,947	3,817	485
1919	1,768	433	4,091	15,742	469	1,052	1,468	21,648	3,700	8,962	603
1920	921	268	2,709	8,893	233	682	1,021	13,188	2,223	5,201	454
1921	1,171	282	3,781	6,082	201	909	1,177	12,625	1,989	5,568	642
1922	2,227	488	5,299	14,271	555	1,254	1,652	24,649	3,983	11,395	749
1923	978	275	2,993	8,319	249	797	1,049	13,603	2,254	5,705	496
1924	1,678	375	4,554	7,446	312	1,134	1,440	17,119	2,765	7,547	715
1925	581	167	1,749	4,661	67	645	821	7,983	1,333	2,197	362
1926	1,271	312	3,244	8,877	234	957	1,231	15,462	2,521	5,805	523
1927	327	113	1,208	3,313	60	462	609	5,354	909	1,796	259
1928	546	164	1,795	3,664	76	579	791	7,496	1,234	2,452	356
1929	1,996	441	4,870	11,911	442	1,209	1,569	22,039	3,521	9,784	715
1930	616	177	1,941	4,019	85	631	841	8,206	1,335	2,678	374
1931	1,340	309	3,462	6,806	202	1,042	1,283	14,509	2,334	5,506	570
1932	586	190	1,971	6,292	147	549	806	9,278	1,570	3,540	367
1933	735	206	2,620	5,933	128	658	915	8,950	1,474	3,609	486
1934	526	187	1,895	6,903	149	466	749	9,111	1,589	3,384	362
1935	583	173	2,174	3,135	95	636	808	7,464	1,238	2,836	424
1936	921	259	2,694	8,993	198	760	1,040	12,695	2,153	4,855	458
1937	381	128	1,463	3,371	73	495	656	6,057	1,002	2,142	301
1938	2,117	436	4,991	9,532	361	1,367	1,649	20,892	3,271	8,812	752
1939	1,627	407	3,752	15,000	413	1,005	1,407	20,158	3,482	8,017	564
1940	794	211	2,346	4,474	107	755	967	9,594	1,570	3,280	433
1941	1,114	280	3,389	9,294	231	865	1,151	12,987	2,215	5,280	576
1942	704	214	2,264	7,440	157	628	895	10,429	1,777	3,948	403
1943	2,061	472	3,881	18,234	425	1,264	1,630	22,773	4,071	8,142	568

Table 4.--Annual maximum discharges as determined from the simulated peaks--Continued

Water year	Station 08074150 Cont.	08074200 Cont.	08074250 Cont.	08074500 Cont.	08074780 Cont.	08074800 Cont.	08074850 Cont.	08075000 Cont.	08075400 Cont.	08075500 Cont.	08075550 Cont.
1944	2,867	532	5,122	14,797	409	1,792	1,958	26,506	4,285	9,792	723
1945	1,760	385	3,548	12,357	247	1,289	1,509	18,943	3,162	6,162	567
1946	1,763	387	4,743	12,969	370	1,171	1,479	18,020	2,964	7,816	738
1947	1,367	343	3,865	9,416	330	907	1,267	17,030	2,743	7,503	606
1948	589	170	2,107	3,288	83	605	817	7,332	1,206	2,647	415
1949	841	223	2,545	4,776	127	758	992	10,223	1,669	3,702	460
1950	1,862	434	4,340	14,041	444	1,124	1,509	21,747	3,597	9,141	638
1951	742	193	2,331	4,416	86	757	937	8,356	1,355	2,867	446
1952	536	181	1,885	6,233	123	488	763	8,811	1,505	3,083	367
1953	665	200	2,320	5,212	140	639	861	9,642	1,558	3,764	422
1954	773	219	1,965	7,716	111	739	961	10,523	1,843	2,980	370
1955	674	195	2,486	3,533	120	616	871	8,442	1,393	3,400	468
1956	419	137	1,629	2,721	65	465	677	5,955	995	2,039	339
1957	953	241	2,769	5,278	134	843	1,071	10,744	1,766	3,890	495
1958	925	247	2,960	8,410	175	805	1,041	11,693	2,031	4,654	516
1959	922	239	2,716	8,010	166	839	1,043	11,470	1,917	4,157	482
1960	2,370	501	5,456	18,771	577	1,491	1,779	24,856	4,296	11,726	769
1961	796	239	2,347	8,495	173	700	954	11,681	2,016	4,058	434
1962	1,000	263	2,919	8,417	178	829	1,087	12,805	2,159	4,581	505
1963	847	216	2,408	5,534	134	818	1,012	10,183	1,667	3,702	446
1964	376	130	1,470	3,218	65	418	634	6,123	1,007	2,058	305
1965	865	224	2,526	6,082	120	795	1,012	10,261	1,668	3,556	457
1966	807	222	2,635	6,463	150	701	962	10,655	1,774	3,926	474
1967	661	173	1,828	3,250	60	756	900	7,444	1,238	2,113	366
1968	1,572	376	4,004	11,007	355	1,021	1,371	18,759	3,061	7,933	610
1969	674	194	2,215	4,465	112	631	874	9,086	1,472	3,265	413
1970	499	165	1,660	5,141	97	531	736	7,966	1,349	2,647	319
1971	977	249	2,922	6,298	152	831	1,080	11,154	1,830	4,234	516
1972	813	225	2,467	5,805	144	702	965	10,902	1,753	3,996	438
1973	1,866	468	4,204	18,572	566	1,040	1,510	22,883	4,069	9,672	607
1974	643	181	2,105	5,501	107	657	864	7,861	1,352	2,746	410
1975	496	171	1,654	5,948	112	463	732	8,289	1,426	2,829	346
1976	2,207	434	4,517	10,503	304	1,513	1,696	21,171	3,356	7,841	677

Table 4.--Annual maximum discharges as determined from the simulated peaks--Continued

Water year	Station 08075650	08075730	08075760	08075770	08075780	08075900	08076000	08076200	08076500	08076700	08077100
1910	2,101	1,237	627	935	166	1,026	2,619	674	2,191	4,671	217
1911	1,766	2,410	399	1,491	359	1,907	2,993	539	1,747	5,788	251
1912	2,875	3,109	597	2,219	494	2,739	4,552	718	2,363	7,924	325
1913	2,405	1,738	705	1,104	211	1,241	3,117	755	2,479	5,634	238
1914	2,618	2,769	615	2,323	570	3,069	4,640	829	2,682	9,999	325
1915	4,116	3,819	814	3,337	754	4,151	6,701	983	3,223	12,199	410
1916	1,996	1,327	574	871	172	1,008	2,312	575	1,867	4,180	211
1917	1,193	1,174	347	639	126	721	1,593	413	1,334	2,835	168
1918	2,450	1,839	691	1,161	247	1,300	3,130	736	2,415	5,505	244
1919	3,411	3,285	721	2,973	708	3,873	5,962	968	3,158	12,285	378
1920	2,006	2,502	446	1,714	413	2,225	3,477	624	2,024	6,966	273
1921	2,865	3,305	573	1,865	379	2,033	3,633	567	1,852	5,124	310
1922	4,475	4,328	841	3,708	838	4,599	7,232	988	3,255	12,613	434
1923	2,192	2,888	511	1,883	448	2,390	3,692	608	1,980	6,827	288
1924	3,715	3,631	743	2,535	524	2,862	4,970	759	2,483	7,530	367
1925	1,301	1,410	410	656	130	790	1,891	547	1,785	4,016	171
1926	2,665	2,345	618	1,835	392	2,248	4,153	762	2,502	7,755	295
1927	801	1,321	286	585	140	748	1,264	366	1,188	2,647	146
1928	1,315	1,352	364	767	156	897	1,878	458	1,501	3,372	184
1929	4,096	3,835	806	3,146	688	3,847	6,384	919	3,035	10,982	401
1930	1,459	1,414	396	841	170	974	2,061	481	1,558	3,723	193
1931	2,900	2,348	677	1,738	345	1,987	3,991	738	2,415	6,661	296
1932	1,385	2,085	347	1,176	293	1,546	2,355	467	1,521	4,728	216
1933	1,860	2,417	412	1,233	260	1,371	2,430	539	1,732	4,444	242
1934	1,250	1,987	289	1,144	295	1,523	2,264	456	1,464	4,879	211
1935	1,508	1,989	400	963	202	1,071	1,968	461	1,492	3,083	210
1936	2,016	2,347	497	1,567	358	1,966	3,313	701	2,264	6,993	262
1937	970	1,552	306	703	165	871	1,488	374	1,216	2,782	165
1938	4,380	3,440	914	2,806	563	3,196	6,056	931	3,052	9,618	392
1939	3,130	2,922	692	2,654	628	3,418	5,460	945	3,062	11,539	355
1940	1,836	1,634	480	1,038	205	1,187	2,491	549	1,790	4,306	221
1941	2,597	2,854	556	1,758	407	2,175	3,520	627	2,022	7,085	296
1942	1,619	2,091	414	1,284	300	1,626	2,703	609	1,966	5,645	232
1943	3,606	2,342	900	2,651	592	3,367	6,064	1,222	3,942	13,793	364

Table 4.--Annual maximum discharges as determined from the simulated peaks--Continued

Water year	Station										
	08075650 Cont.	08075730 Cont.	08075760 Cont.	08075770 Cont.	08075780 Cont.	08075900 Cont.	08076000 Cont.	08076200 Cont.	08076500 Cont.	08076700 Cont.	08077100 Cont.
1944	5,156	2,568	1,233	2,942	562	3,472	7,488	1,346	4,460	14,091	404
1945	3,210	2,397	888	1,928	394	2,289	4,961	1,101	3,600	10,646	309
1946	3,897	3,777	769	2,639	610	3,267	5,119	820	2,653	9,576	378
1947	2,987	3,499	593	2,451	560	3,047	4,804	716	2,355	8,357	342
1948	1,506	1,747	374	878	176	964	1,914	413	1,328	3,023	202
1949	1,959	2,310	482	1,188	241	1,369	2,701	551	1,795	4,568	237
1950	3,666	3,365	762	2,964	677	3,760	6,099	959	3,138	11,727	382
1951	1,807	1,601	474	919	172	985	2,191	493	1,596	3,468	212
1952	1,265	1,851	311	1,000	243	1,310	2,177	472	1,519	4,659	200
1953	1,618	2,243	409	1,227	279	1,495	2,534	538	1,761	4,503	229
1954	1,595	1,182	487	916	192	1,147	2,507	703	2,269	6,018	200
1955	1,731	2,371	384	1,166	248	1,301	2,279	426	1,367	3,484	233
1956	1,096	1,471	286	683	145	779	1,486	349	1,115	2,527	171
1957	2,192	1,974	541	1,265	253	1,418	2,866	588	1,912	4,724	250
1958	2,194	2,609	541	1,540	331	1,801	3,132	761	2,463	6,683	270
1959	2,118	2,025	538	1,337	304	1,681	2,962	669	2,154	6,086	249
1960	4,538	4,662	1,071	3,861	880	4,790	7,318	1,396	4,520	14,795	442
1961	1,771	1,801	442	1,315	308	1,695	2,936	626	2,021	6,400	237
1962	2,279	2,240	531	1,463	325	1,809	3,263	716	2,328	6,897	263
1963	1,939	1,909	524	1,180	256	1,433	2,641	573	1,863	4,852	227
1964	968	1,376	256	659	145	796	1,498	344	1,114	2,754	162
1965	1,990	1,771	506	1,139	225	1,284	2,678	570	1,852	4,590	233
1966	1,938	2,261	442	1,301	282	1,564	2,751	548	1,784	5,350	246
1967	1,508	999	483	654	117	708	1,829	532	1,711	3,296	178
1968	3,264	3,228	675	2,556	572	3,173	5,266	827	2,715	9,559	351
1969	1,617	1,837	396	1,051	222	1,232	2,349	481	1,558	4,090	216
1970	1,165	1,439	339	849	195	1,076	1,945	489	1,577	4,040	184
1971	2,270	2,238	532	1,396	282	1,574	3,015	583	1,888	4,945	262
1972	1,862	1,985	447	1,258	271	1,529	2,854	557	1,816	5,184	238
1973	3,499	3,517	730	3,320	829	4,355	6,308	1,028	3,305	13,485	399
1974	1,572	1,574	414	892	217	1,170	1,979	461	1,478	4,150	203
1975	1,141	1,658	297	917	222	1,202	2,024	458	1,469	4,399	190
1976	4,282	2,476	1,016	2,386	453	2,735	5,929	1,075	3,537	10,446	360

construction of roads, buildings, plants, shopping centers, and parking lots. Second, the natural condition of the channel is altered by channel-improvements, which may mean that the channel is cleaned of vegetation, the channel is concrete lined, or the channel is replaced by a storm-sewer system. All methods of improvement provide a more hydraulically efficient cross-sectional shape. Basin characteristics selected for this study attempted to quantify these changes in a basin.

The basin characteristics used are (1) drainage area, (2) bank-full channel conveyance, and (3) percentage of urban development. Drainage area, A, in square miles, is defined as the total contributing drainage area at the gaging-station location. It is determined by planimetering the delineated area on topographic maps.

Bank-full channel conveyance, K, is defined as the value computed for conveyance at the controlling section, when the stage elevation is equal to that of the lower bank, using Manning's equation for open-channel flow:

$$K = \frac{1.49}{n} A_x R^{\frac{2}{3}}$$

where A_x = bank-full cross-sectional area, in feet squared;

R = bank-full hydraulic radius, in feet; and

n = Manning's roughness coefficient.

This measure of conveyance is an indication not only of channel capacity but also of the relative efficiency of a channel. A_x and R are determined by field surveys at the controlling section of a channel for the gaging station. Manning's *n* is selected in the field by experienced personnel.

Johnson and Sayre (1973, p. 44) noted:

"The estimated T-year discharges from the relationship are design values. Sufficient channel capacity must be provided or inundation, resulting from temporary ponding, will occur in parts of the basin. Unless adequate channel capacity is provided, flooding can occur at low points along the channel as a result of channel flooding or at street intersections and grade separations that cannot be adequately drained.

"Because the discharges are design values, they should not be used to predict water-surface elevations along a channel that has a capacity less than the selected T-year discharge indicated by the relation."

These statements emphasize the importance of channel capacity in determining flood-peak magnitude for this coastal area.

Johnson and Sayre (1973, p. 5) also noted the increase in the magnitude of flood peaks following channel improvements. Espey and Winslow (1968, p. 55) discussed the effects of changes in channel conveyance on the time of rise and on the unit-peak discharge of streams in the Houston area. Changes in the magnitude of peak discharges in the Houston area can be largely attributed to channel improvements. This study has shown channel conveyance to be a more important parameter in estimating the magnitude and frequency of floods in Houston than other commonly used indices of urbanization.

The percentage of urban development, A_D, is defined as the percentage of the total contributing drainage area within 200 feet of streets, roads, parking lots, and industrial sites that is drained by open street ditches or storm sewers. A_D is highly correlated with the percentage of impervious area in a basin, I, as used by Johnson and Sayre (1973), but is much easier to determine accurately from aerial photographs.

The values for A, K, and A_D, as determined for this study, are given in table 5.

FLOOD-FREQUENCY ANALYSIS

The flood-frequency characteristics as computed from the observed annual-peak discharge data are given in table 6; the observed station skew was applied to the values because of a lack of information concerning skew variation in an urbanizing coastal area. Low outliers that caused an abnormal negative bias to the annual-peak discharge data were discounted according to the Water Resources Council (1977) guidelines.

The flood-frequency characteristics computed from the simulated annual-peak discharge data are given in table 7. The skew value determined from the 67-year series of simulated discharges at each station was used in computing these characteristics.

The determination of regional relationships for predicting flood-frequency characteristics requires one set of flood-frequency values (dependent variables) for each of the sites in the study area. Water Resources Council (1977) guidelines suggest that flood-frequency curves determined from observed data may be adjusted by the use of simulated flood-frequency values, but the guidelines require that any adjustments incorporate the relative accuracy of the simulated and observed data.

Several methods of adjusting or weighting the flood-frequency curves were evaluated. These methods were (1) averaging, (2) weighting based on length of observed record, and (3) weighting based on error characteristics.

In the averaging method, the weighted values are obtained by averaging the results from the observed and simulated data for each recurrence interval. This procedure is based on the assumption that each data series represents conditions that are equally likely to occur at a site under the stated degree of urbanization.

The weighting method based on the length of observed record requires a specified amount of observed data to define a flood peak for a particular recurrence interval. If this amount of observed data is available, then the weighted flood-frequency value is equal to the observed value. Otherwise, the weighted flood-frequency value is equal to some combination of the observed value and the simulated value. In this method, the observed data are weighted more heavily than the simulated data.

In the method of weighting based on error characteristics, the weighted values are determined by considering the relative errors present in each data series. The procedure for computing relative error is analogous to a variance

Table 5.--Selected basin characteristics for sites in study

Station	A	K	A_D (percent)
08074150	8.81	1.7×10^5	54.0
08074200	2.56	2.1×10^4	54.7
08074250	11.4	2.3×10^5	77.5
08074500	84.7	1.7×10^6	57.7
08074780	7.87	3.5×10^4	44.9
08074800	12.0	5.6×10^4	55.7
08074850	4.29	8.2×10^4	88.3
08075000	88.4	2.8×10^6	64.4
08075400	20.2	2.8×10^5	69.3
08075500	64.0	5.3×10^5	73.7
08075550	2.87	3.6×10^4	71.8
08075650	10.1	4.5×10^5	85.3
08075730	8.21	2.0×10^5	89.4
08075760	2.75	5.1×10^4	98.9
08075770	14.7	2.7×10^5	95.0
08075780	8.73	1.2×10^4	47.2
08075900	36.1	8.9×10^4	37.0
08076000	69.6	2.9×10^5	43.9
08076200	8.69	3.1×10^4	52.8
08076500	28.3	1.0×10^5	74.1
08076700	182.0	9.3×10^5	60.6
08077100	1.33	2.0×10^4	93.2

Table 6.--Flood-frequency characteristics determined from observed data

Station	Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}	Annual peak data: Mean of logs	Annual peak data: Standard deviation	Skew
08074150	750	1,290	1,650	2,110	2,430	2,750	2.8490	0.304	-0.520
08074200	220	345	420	500	545	590	2.3000	.284	-.890
08074250	1,930	3,640	4,820	6,260	7,260	8,210	3.2430	.372	-.700
08074500	7,920	12,090	14,880	18,400	21,000	23,600	3.8890	.227	-.270
08074780	175	335	485	735	970	1,250	2.2630	.323	.370
08074800	700	1,020	1,230	1,490	1,690	1,880	2.8380	.200	-.190
08074850	1,040	1,130	1,160	1,190	1,210	1,220	3.0090	.050	-1.02
08075000	12,520	19,230	24,030	30,430	35,420	40,600	4.0960	.222	-.030
08075400	2,210	3,200	3,830	4,600	5,150	5,680	3.3340	.200	-.310
08075500	4,350	7,220	9,270	11,980	14,040	16,140	3.6260	.273	-.260
08075550	425	585	685	815	910	1,000	2.6280	.164	-.060
08075650	1,550	2,890	3,960	5,540	6,850	8,280	3.1850	.325	-.100
08075730	2,260	2,980	3,410	3,920	4,280	4,620	3.3480	.148	-.250
08075760	445	600	695	820	910	995	2.6460	.155	-.070
08075770	1,240	2,300	3,000	3,820	4,390	4,900	3.0440	.370	-.780
08075780	330	455	535	625	685	745	2.5050	.179	-.360
08075900	2,140	2,690	2,990	3,300	3,500	3,670	3.3180	.132	-.600
08076000	3,350	5,010	6,210	7,810	9,080	10,400	3.5270	.206	.070
08076200	650	890	1,040	1,210	1,330	1,440	2.8010	.174	-.360
08076500	2,050	2,720	3,130	3,620	3,970	4,310	3.3070	.150	-.190
08076700	9,650	14,650	18,060	22,430	25,710	29,010	3.9780	.221	-.180
08077100	245	360	430	515	570	625	2.3770	.209	-.420

Table 7.--Flood-frequency characteristics determined from 67 years of simulated data

Station	Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}	Simulated annual peak data: Mean of logs	Simulated annual peak data: Standard deviation	Skew
08074150	935	1,450	1,840	2,390	2,840	3,320	2.9760	0.223	0.160
08074200	250	350	420	515	585	665	2.4060	.167	.220
08074250	2,720	3,700	4,360	5,210	5,840	6,500	3.4390	.158	.080
08074500	6,880	10,600	13,400	17,300	20,530	23,960	3.8440	.218	.180
08074780	165	290	400	560	705	870	2.2320	.282	.260
08074800	795	1,050	1,220	1,430	1,580	1,740	2.9030	.141	.100
08074850	1,050	1,330	1,520	1,750	1,930	2,100	3.0260	.112	.250
08075000	11,800	16,800	20,500	25,500	29,500	33,700	4.0760	.180	.250
08075400	1,930	2,770	3,380	4,220	4,880	5,590	3.2940	.179	.270
08075500	4,320	6,650	8,430	10,950	13,030	15,300	3.6440	.216	.250
08075550	480	595	665	750	810	870	2.6800	.111	.010
08075650	2,110	3,060	3,730	4,620	5,310	6,020	3.3280	.190	.080
08075730	2,170	2,920	3,420	4,050	4,530	5,000	3.3380	.153	.060
08075760	510	695	820	990	1,120	1,250	2.7130	.155	.190
08075770	1,370	2,120	2,690	3,510	4,180	4,910	3.1450	.219	.230
08075780	305	480	615	810	975	1,150	2.4890	.231	.200
08075900	1,680	2,640	3,390	4,470	5,360	6,350	3.2330	.228	.240
08076000	3,070	4,590	5,720	7,310	8,600	9,980	3.5000	.201	.250
08076200	630	840	990	1,190	1,340	1,500	2.7960	.141	.280
08076500	2,060	2,770	3,260	3,930	4,450	5,000	3.3120	.146	.300
08076700	5,760	8,710	10,900	14,000	16,600	19,300	3.7690	.207	.240
08077100	255	325	370	430	470	515	2.4080	.123	.200

analysis except that the expected value of the mean square error is used as an indicator of error instead of the regression variance. Application of this method produced flood-frequency values which were more heavily weighted toward the simulated data at lower recurrence intervals. This result was the opposite of those found on rural watersheds (Wibben, 1976; Thomas and Corley, 1977; Curtis, 1977; Olin and Bingham, 1977).

The method of averaging the results from the observed and simulated data for each recurrence intervals was used to produce the Q_t values used as dependent variables (table 8). From a practical standpoint, however, there is little difference between the three weighting methods. Comparisons between the various weighted values of Q_t showed that the values produced for Q_t by the other methods were always within +21.5 and -16.8 percent of the Q_t values obtained by the averaging method. More than 93 percent of the Q_t values produced by the other methods were within $\pm$10 percent of the values used.

Comparisons of the results of predictions based on equations developed from the various sets of weighted Q_t values showed that regardless of the method of weighting used the predicted Q_t values were within +10.9 and -8.7 percent of the final predicted Q_t values. Only 1.3 percent of the Q_t values predicted from equations based on the other weighting methods differed by more than $\pm$7.5 percent from the final predicted values.

DEVELOPMENT OF REGRESSION RELATIONSHIPS

Multiple-regression techniques were used to define regional relationships to predict flood-peak magnitudes from drainage-basin characteristics. The dependent variables are the flood-frequency values in table 8. The independent variables are given in table 5. The value of the variables were transformed to base 10 logarithms prior to performing the regression analysis.

Several regression models involving these independent variables were investigated. These included the following forms:

$$\text{(a)} \quad Q_t = aA^{b_1}K^{b_2}A_D{}^{b_3}$$

$$\text{(b)} \quad Q_t = aA^{b_1}K^{b_2}$$

and

$$\text{(c)} \quad Q_t = aA^{b_1}\,[K(1.0 + 0.01A_D)]^{b_2}$$

where Q_t = discharge in cubic feet per second for a recurrence interval of t years;

A,K,A_D = the independent variables as defined previously;

b_1,b_2,b_3 = regression coefficients; and

a = regression constant.

The Q_t values predicted from these three forms were compared for the sites of this study. The difference between the various predicted Q_t values was always within $\pm$8 percent and were within $\pm$5 percent over 98 percent of the time.

Table 8.--Flood-frequency characteristics used as dependent variables

Sequence	Station	Flood-peak discharge (cubic feet per second)					
		Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}
1	08074150	840	1,370	1,740	2,250	2,640	3,040
2	08074200	235	350	420	505	565	625
3	08074250	2,320	3,670	4,590	5,740	6,550	7,360
4	08074500	7,400	11,340	14,140	17,850	20,760	23,780
5	08074780	170	315	440	650	835	1,060
6	08074800	745	1,040	1,220	1,460	1,640	1,810
7	08074850	1,040	1,230	1,340	1,470	1,570	1,660
8	08075000	12,160	18,020	22,260	27,960	32,460	37,150
9	08075400	2,070	2,980	3,600	4,410	5,020	5,640
10	08075500	4,340	6,940	8,850	11,460	13,540	15,720
11	08075550	455	590	675	785	860	935
12	08075650	1,830	2,980	3,840	5,080	6,080	7,150
13	08075730	2,220	2,950	3,420	3,980	4,400	4,810
14	08075760	475	645	760	905	1,010	1,120
15	08075770	1,300	2,210	2,840	3,660	4,280	4,900
16	08075780	315	470	575	720	830	950
17	08075900	1,910	2,660	3,190	3,880	4,430	5,010
18	08076000	3,210	4,800	5,960	7,560	8,840	10,190
19	08076200	640	865	1,010	1,200	1,340	1,470
20	08076500	2,060	2,740	3,200	3,780	4,210	4,660
21	08076700	7,700	11,680	14,480	18,220	21,160	24,160
22	08077100	250	340	400	470	520	570

Form (a) was unsatisfactory because the variable A_D did not remain statistically significant at the 5-percent significance level. Form (c) was formulated in an attempt not only to include the measure of urban development, A_D, but also to reduce the standard error of estimate resulting from form (b). Form (c) was developed by considering that the flood peak from an urbanized area in Houston was proportional to the product of bank-full channel conveyance, K, and a factor representing the amount of urban development. The factor, $1.0 + 0.01A_D$, was selected on the basis of work done by Carter (1961). Because the standard error of estimate for form (c) was an improvement over that of form (b), form (c) was used for this study. Form (c) was evaluated for bias at the 5- and 50-year recurrence intervals. None was apparent.

The values determined for the regression constant and the regression coefficients are given in table 9. Station 08074780, Keegans Bayou at Keegans Road, was not used in the development of these relationships because of missing data. The variation of the constant and coefficients indicate consistency and continuity with the regression model. Table 9 also presents the standard error of estimate (Se) and the multiple-correlation coefficent (MCC) for the regional relationship at each recurrence interval.

The results of applying the relationships in table 9 to the stations used for this study are given in table 10. The flood-frequency values in table 10 should be considered as good estimates as long as conditions in the basin remain similar to those of December 1976. The effect of changes in a basin may be predicted by use of the relationships given in table 9.

APPLICATION OF REGRESSION RELATIONSHIPS

The regression relationships are shown in table 9. They provide a method for computing flood-peak magnitudes for recurrence intervals of 2, 5, 10, 25, 50, 100, and 500 years on ungaged and unregulated streams in the Houston, Texas, metropolitan area. These sites may require flood-frequency information for a variety of reasons. For example, the site may be a completely urbanized basin for which information is required to establish flood insurance rates; or the site may be completely undeveloped, but information is required by developers to determine predicted future flooding. In the first case, selection of the basin characteristics may be fairly straightforward, however, the second case can illustrate several possible errors in selecting basin characteristics. This section provides guidelines for computing the basin characteristics.

Drainage Area

The drainage area should be delineated on a U.S. Geological Survey 7-1/2-minute or 15-minute topographic map, with the aid of field reconnaissance. In addition, the drainage basin should be inspected so that drainage ditches, which are not shown on the map, but which may cause variations in the total contributing drainage area, can be located. The field inspection may also discover other features, such as detention storage and storm sewers that will affect the total contributing drainage area.

Table 9.--Regionalized relationships for Q_t

t (years)	Regression constant	Regression coefficient for A	Regression coefficient for $K(1.0+0.01A_D)$	S_e (percent)	MCC
2	2.028	0.383	0.447	25.1	0.978
5	2.208	.392	.468	19.7	.987
10	2.301	.399	.478	18.1	.989
25	2.460	.410	.487	17.1	.991
50	2.576	.419	.492	16.9	.991
100	2.710	.428	.495	17.1	.991
500	3.097	.451	.498	18.1	.991

Example: $Q_2 = 2.028\ A^{0.383}[K(1.0 + 0.01A_D)]^{0.447}$

Table 10.--Predicted flood-peak discharges for recurrence intervals of 2, 5, 10, 25, 50, and 100 years

Sequence	Station	Flood-peak discharge (cubic feet per second)					
		Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}
1	08074150	1,230	1,780	2,130	2,610	2,970	3,310
2	08074200	300	415	480	570	635	695
3	08074250	1,660	2,420	2,920	3,600	4,120	4,600
4	08074500	8,300	12,830	15,990	20,510	24,070	27,550
5	08074780	565	790	930	1,120	1,260	1,400
6	08074800	850	1,200	1,430	1,740	1,970	2,190
7	08074850	740	1,050	1,240	1,500	1,690	1,870
8	08075000	10,740	16,800	21,060	27,160	31,970	36,670
9	08075400	2,210	3,250	3,940	4,900	5,630	6,330
10	08075500	4,620	6,970	8,580	10,860	12,650	14,400
11	08075550	420	585	685	815	910	1,000
12	08075650	2,180	3,230	3,920	4,860	5,560	6,220
13	08075730	1,420	2,060	2,470	3,040	3,460	3,850
14	08075760	515	725	850	1,020	1,140	1,260
15	08075770	2,050	3,010	3,650	4,530	5,190	5,820
16	08075780	370	500	585	700	785	865
17	08075900	1,500	2,160	2,600	3,210	3,680	4,140
18	08076000	3,350	4,970	6,080	7,650	8,880	10,090
19	08076200	570	795	935	1,130	1,270	1,410
20	08076500	1,610	2,320	2,790	3,460	3,960	4,450
21	08076700	8,560	13,170	16,400	21,110	24,870	28,610
22	08077100	255	345	400	475	525	570

Bank-Full Channel Conveyance

The term "controlling reach" as used in this report refers to that reach of a channel, downstream from the site in question, in which the frictional resistance of the streambed and banks determine the rate of flow at a given stage. Because flat slopes and relatively slow water velocities are characteristic of streams in the Houston area, reach control is typical except at extremely low stages. Reach control must exist at bank-full stage if the technique described in this report is to be applied. Before determining bank-full channel conveyance, then, the reach downstream from the site should be inspected to insure that there are no dams, culverts, or other physical features that could cause an appreciable break in the water-surface slope.

The regional relationships shown in table 9 may be used as aids in the design of channels to carry future flood waters. When the relationships are used for this purpose, some guidelines are required with respect to the magnitude of channel conveyance. A relationship of maximum-allowable channel conveyance (K_{MAX}) to drainage area as shown on figure 2 was determined by plotting the values for K and A for all sites used in the study. The assumption was made that none of the channels were adequately designed to carry the maximum discharges likely to occur when the basins are completely urbanized.

Figure 2 was developed by using a straight-line relationship through points 5 percent greater than those having the greatest ratio of K to A. The user should be aware that K_{MAX} is not intended to be an optimal value. It should be considered a boundary for which the value of K substituted into the relationships developed by this study cannot exceed. If the computed K is greater than K_{MAX}, then K_{MAX} should be used. However, if the computed K is less than or equal to K_{MAX}, the assumption should not be made that the designed channel conveyance is acceptable for future development without considering other economic and engineering variables.

Bank-full channel conveyance may be determined at existing sites as follows: (1) Obtain two or more representative cross sections of the channel in the controlling reach; (2) select the elevation of the top of the lowest bank as bank-full stage for each cross section; (3) select Manning's n and compute the bank-full conveyance, K, for each cross section; and (4) determine the arithmetic mean value of K for the site.

Percentage of Urban Development

The percentage of urban development, A_D, may be determined by the use of aerial photographs. After delineating the total contributing drainage area on the photographs, the 200-foot boundaries for urbanized areas within the drainage basin may be easily marked either on the photographs or on a transparent overlay. If aerial photographs are not available, field reconnaissance will yield current conditions. The developed area as delineated may be measured in square miles by planimetering. This value is then converted to a percentage of the total contributing drainage area.

In summary, the relationships are applied at ungaged and unregulated sites in the Houston, Texas, metropolitan area by the following procedure: (1) Locate the site on a map; (2) determine the total contributing drainage

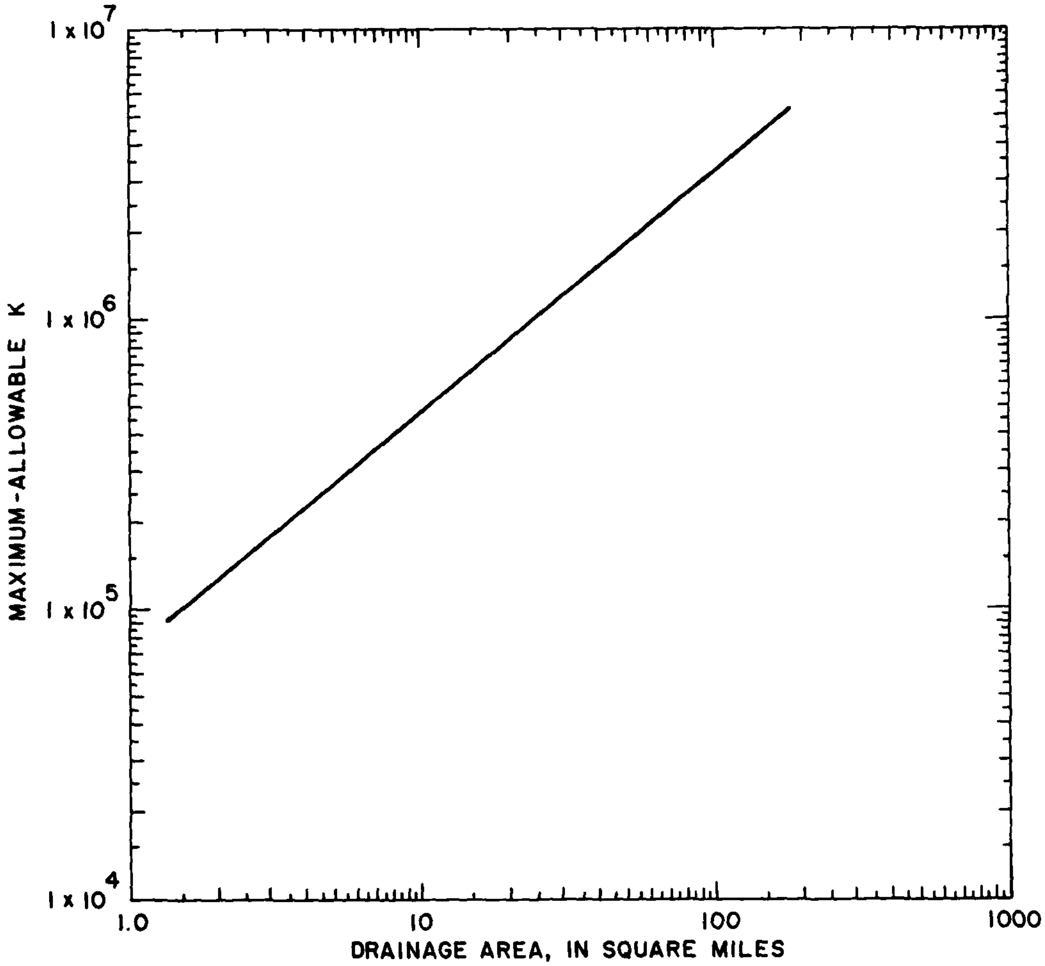


FIGURE 2.-Maximum allowable conveyance versus drainage area

area; (3) define the urbanized area for the date required; (4) compute bank-full channel conveyance for the controlling reach of the channel downstream from the site; and (5) compute flood-frequency values by the use of the regression equations in table 9.

The technique may also be used to evaluate the effects of future development. Thus, various stages of urbanization may be assumed to predict urban flood-frquency characteristics at these different stages of development.

Limitations and Special Cases

The technique presented in this report does not apply to regulated streams. Flood-frequency estimates are defined for unregulated streams in the Houston area within the following limits of basin characteristics.

(1)	Contributing drainage area, A	1.33 to 182 square miles
(2)	Bank-full channel conveyance, K	1.2×10^4 to 2.8×10^6
(3)	Percentage of urban development, A_D	37.0 to 98.9

The following are some special cases in which the technique may be used: (1) Use of the technique to predict changes due to urbanization; (2) use of the technique at a site where bank-full channel conveyance has an abrupt change; and (3) use of the technique in a design situation. These are discussed below.

The use of the technique to predict changes due to urbanization is easily accomplished if the user wants to compare current conditions with conditions of complete urbanization. The percentage of urban development, A_D, is increased to 100 and K_{MAX} is determined from figure 2. The flood frequency estimates obtained by using these values with the relationships in table 9 will indicate the effects of complete urbanization and complete rectification of a channel.

If the user wants to compare a site in its completely rural state to a completely urbanized state, some assumptions must be made. Values for A_D at several rural sites in the Houston area were computed at between 15 and 25 percent. Thus, A_D for rural sites is chosen as 20 percent. Assuming that K_{MAX} may be selected from figure 2, the bank-full channel conveyance for a completely unrectified (rural) channel was chosen by assuming that Halls Bayou at Deertrail Street and Greens Bayou at U.S. Highway 75 were typical of such streams, for which the average ratio of K_{MAX} to K_{RURAL} equals 15. This value is assumed to be near the upper limit; therefore, the effects of urbanization on flood-frequency characteristics should not be greater than those given in table 11. At any site in the Houston area, a change from completely rural to completely urban conditions should not increase the peak discharge of a 100-year flood by more than a factor of 4.9.

An abrupt change in channel conveyance may occur at a site where channel rectification has halted. Use of the technique at such a site requires the computation of flood-frequency characteristics for both the larger and smaller values of K. The flood-frequency characteristics computed by using the smaller K would be valid for the period that the abrupt change in the channel remains. Once the channel rectification is complete, the flood-

Table 11.--Effects of urbanization on flood-frequency characteristics

Case no.	Description of change due to urbanization	Factor by which flood peak increases					
		Recurrence interval (years)					
		2	5	10	25	50	100
1	Development increases from completely rural to completely urban	1.26	1.27	1.28	1.28	1.29	1.29
2	Channel is changed from completely unrectified (rural) to completely rectified state	3.36	3.55	3.65	3.74	3.79	3.82
3	Cases 1 and 2 occur together	4.23	4.51	4.67	4.78	4.89	4.93

frequency characteristics computed by using the larger K would be valid for such a site, assuming the larger K did not exceed K_{MAX}.

If the technique is being used to estimate Q_t values for a channel design, the value of K used for the planned channel should not produce unreasonably large estimates of Q_t. The following procedure is suggested to check the value of K: (1) From the design configuration of the planned channel, compute K; (2) determine A for the basin; and (3) enter figure 2 with the value of A and determine K_{MAX}. If K is greater than K_{MAX}, then K_{MAX} must be used in determining the Q_t values. If K is equal to or less than K_{MAX}, the computed K must be used in these relationships. Whether or not the designed channel should be built may be determined by other considerations.

The user of the equations in table 9 is cautioned that the use of a K value larger than K_{MAX} may result in the computation of peak discharges larger than the basin can produce. This problem should be avoided by limiting the maximum value of K substituted into these relationships to K_{MAX} (fig. 2). Also, the user should never confuse the bank-full channel conveyance value computed for this technique with the actual channel conveyance required by various flows.

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